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May 6, 2005

Mr. Edgar Pejoro
Regency Centers
555 South Flower Street, Suite 3500
Los Angeles, California 90071

Subject: **Report of Geotechnical Investigation and Seismic Hazard Evaluation Study**
Proposed Shopping Center
3759-3763 State Street
Santa Barbara, California
Arroyo Geotechnical Project No. 12184-2000

Dear Mr. Pejoro:

This report presents the results of our geotechnical investigation and seismic hazard evaluation study for the site of a proposed shopping center in Santa Barbara, California. The purpose of this study was to evaluate the surface and subsurface geotechnical conditions and develop geotechnical recommendations for project design. The work was performed in accordance with our revised proposal dated March 8, 2005 and your authorization dated March 18, 2005.

We appreciate the opportunity to assist you and look forward to future projects. If you have any questions, please do not hesitate to contact us.

Respectfully submitted,

ARROYO GEOTECHNICAL

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EXHIBIT H

ARROYO GEOTECHNICAL

REPORT
GEOTECHNICAL INVESTIGATION AND SEISMIC EVALUATION STUDY PROPOSED SHOPPING CENTER 3759-3763 STATE STREET SANTA BARBARA, CALIFORNIA

Prepared for

Regency Centers
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Prepared by

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Arroyo Geotechnical Project No. 12184-2000

May 6, 2005

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1.0 INTRODUCTION

1.1 PURPOSE

This report presents the results of a geotechnical investigation performed by Arroyo Geotechnical (Arroyo) for the site of a proposed shopping center in Santa Barbara, California. The purpose of this investigation was to evaluate the surface and subsurface soil conditions and develop geotechnical recommendations for project design.

1.2 PROJECT SITE AND PROPOSED CONSTRUCTION

The project site is located in a commercial area with the State Street and Hitchcock Way as its northern and eastern boundaries, and by two natural creeks, Barger Canyon Creek and San Roque Creek, along the western and southern boundaries, respectively. The project site is shown on Figure 1, Site Location Map.

The proposed construction will include following five new buildings and associated driveway and parking facilities:

- Whole Foods Building "A", 40,000 sf. with Roof Parking;
- Circuit City Building "B", 20,577 sf.;
- Shops "C", 3,000 sf.;
- Citibank Building "D", 4,000 sf.;
- Retail Building "E", 1,300 sf.;
- On grade Parking, Driveways, and Truck Loading Ducks.

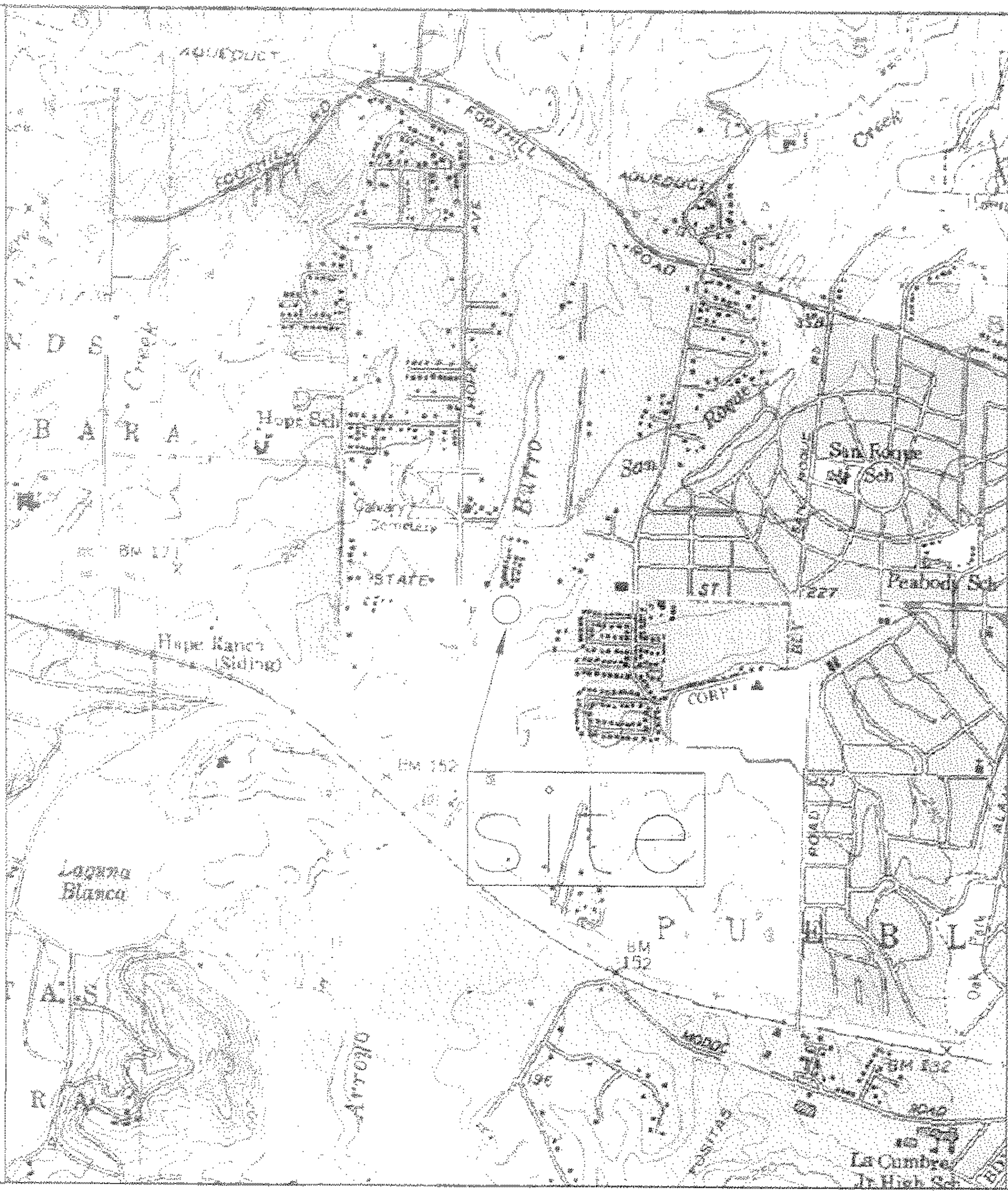
We were informed that the design loads would be in the order of 5 kips per lineal foot for perimeter exterior walls and 160 kips for interior concentrated columns for the Whole Food Building "A." The design loads for the remaining structures would be 3.5 kips per lineal foot and 90 kips for the exterior walls and interior columns, respectively.

1.3 SCOPE OF WORK

The scope of geotechnical services performed for this project included the following:

- Research and review of published and unpublished geologic and geotechnical documents;
- Field exploration consisting of drilling, sampling and logging nineteen exploratory borings;
- Geotechnical laboratory testing of representative bulk and relatively undisturbed soil samples;

- Geotechnical and seismic hazard analyses to develop design and construction recommendations;
- Stability analysis of slope banks along Barger Canyon Creek and San Roque Creek; and
- Preparation of this report presenting our findings, conclusions and recommendations.



Base map from U.S.G.S. 7.5 Minute Series (Topographic)
Santa Barbara Quadrangle, California

N

0 1700 3400 FEET

0 0.25 0.5 1.0 KILOMETER

ARROYO GEOTECHNICAL

SITE LOCATION MAP

Figure 1

PROPOSED SHOPPING CENTER IN SANTA BARBARA

Project No. 12184-2000

Date: 04-29-2005

2.0 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 AVAILABLE GEOTECHNICAL INFORMATION

A due diligence study was performed by Leighton Consulting, Inc. (Leighton) for the proposed development. Seven soil borings (B-1 through B-7) were drilled to depths varying from 21.5 to 71.5 feet on April 29 and 30, 2004 for this study. Approximate locations of these borings are shown in Figure 2. A summary report was prepared by Leighton (2004), which includes preliminary boring logs of Borings B-1 through B-3. At start of this project, Arroyo received preliminary boring logs of Borings B-4 through B-7. Boring information, including boring number, ground surface elevation, borehole depth and groundwater level measurement, is summarized in Table 1. A copy of these boring logs is presented in Appendix A. Arroyo reviewed soil data contained in these boring logs and planned a field investigation program for this project.

2.2 FIELD INVESTIGATION

Field investigation included a site reconnaissance and subsurface exploration. During the reconnaissance, surface conditions were noted, and locations of borings were determined.

Nineteen exploration borings were drilled on April 4 through 6, 2005. Exploration information is also presented in Table 1. Approximate boring locations are shown on Figure 2, Boring Location Map. The boring logs are presented in Appendix B.

All borings (A-1 through A-19) were drilled using truck-mounted drill rig (CME 75) equipped with 8-inch diameter hollow-stem augers. Soils were continuously logged and classified in the field by an experienced staff in accordance with the Unified Soil Classification System. Field descriptions have been modified, where appropriate, to reflect laboratory test results.

Relatively undisturbed ring samples were obtained using the California split-spoon (drive) sampler, which has an outside diameter of 3.25 inches and is lined inside with 2.42-inch inside diameter 1-inch long brass rings. Soil samples were also obtained from the Standard Penetration Test (SPT) split-barrel sampler, which has an outside diameter of 2 inches and an inside diameter of 1.4 inches. The soil samples were collected for laboratory tests at frequent intervals of depth, alternating between the California sampler and the SPT sampler. Both samplers were driven with a 140-lb automatic trip hammer falling a distance of 30 inches, 12 inches (or refusal) into the ground for the drive sampler and 18 inches (or refusal) into the ground for the SPT sampler. The numbers of blow to advance the sampler each 6 inches or less of penetration were recorded and are shown on the boring logs in Appendix B. The central portions of the ring samples were carefully sealed in waterproof plastic containers for shipment to the Arroyo Geotechnical laboratory. In addition, bulk samples of the near surface soils were collected for laboratory tests.

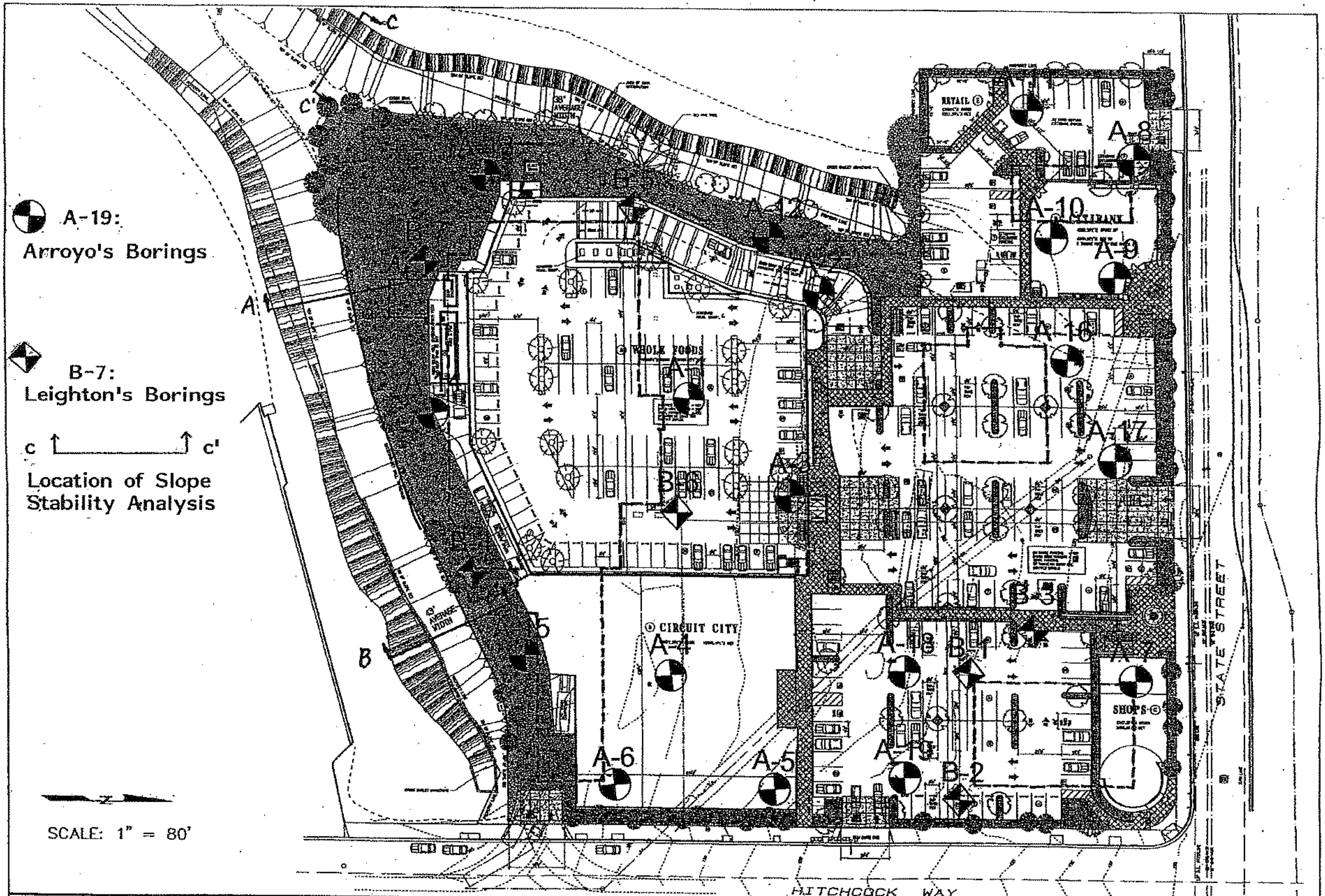
TABLE 1. SOIL EXPLORATION INFORMATION

Boring No.	Approximate GSE (ft)	Depth (ft bgs)	Depth to GW Table (ft bgs)
		Leighton's Borings	
B-1	+91	51.5	19.5
B-2	+92	21.5	19
B-3	+93	21.5	Not Encountered
B-4	+87	31.5	26
B-5	+91	41.5	32
B-6	+92	31.5	Not Encountered
B-7	+89	71.5	30
		Arroyo's Borings	
A-1	+92	51.5	29
A-2	+92	31.0	30
A-3	+92	31.5	29
A-4	+92	51.0	30
A-5	+92	31.5	Not Encountered
A-6	+92	31.0	Not Encountered
A-7	+97	31.5	30
A-8	+96	31.0	30
A-9	+96	31.5	30
A-10	+96	51.0	30
A-11	+95	31.0	Not Encountered
A-12	+91	26.0	Not Encountered
A-13	+90	26.0	Not Encountered
A-14	+89	26.0	Not Encountered
A-15	+87	26.0	Not Encountered
A-16	+94	11.0	Not Encountered
A-17	+95	11.0	Not Encountered
A-18	+90	11.0	Not Encountered
A-19	+91	11.0	Not Encountered

Notes:

(1) GSE = Ground Surface Elevation; bgs = below ground surface; GW = Groundwater

(2) Depth to GW table was measured during subsurface exploration. The GW table may fluctuate and may be higher than those listed in this table.



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PROPOSED SHOPPING CENTER IN SANTA BARBARA

Project No.: 12184-2000

Date: 05-03-2005

BORING LOCATION MAP

Figure 2

2.3 LABORATORY TESTING

Laboratory tests were conducted on selected samples of the earth materials to determine their physical properties and engineering characteristics. The number and selection of particular tests were based on the geotechnical conditions on the project site and the proposed construction. The following laboratory tests were performed:

- In-situ Moisture Content and Dry Density;
- Percent Passing #200 Sieve;
- Sieve and Hydrometer Analysis;
- Atterberg Limits;
- Direct Shear;
- Consolidation;
- Permeability;
- Expansion Index;
- R-Value; and
- Soil Corrosivity (Minimum Resistivity, pH, Sulfate Content and Chloride Content).

The laboratory tests were conducted in general accordance with American Society for Testing and Materials (ASTM) Standards or California Test Methods. In situ-moisture content and dry density test results are shown on the boring logs. The remaining laboratory test results are provided in Appendix C.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site is currently occupied by several commercial structures. There are paved parking lots between these structures and several paved driveways to the surrounding streets. The proposed building areas are approximately flat.

3.2 SUBSURFACE CONDITIONS

The subsurface soils consist of predominantly silty clay and sandy silt interbedded with silty sand at various depths and having various thicknesses.

3.3 GROUNDWATER

We observed groundwater at a depth of 29 to 30 feet during our subsurface exploration in April 2005. The LCI's boring logs indicate the depth to groundwater varied from 19 to 32 feet in April 2004.

3.4 SOIL EXPANSION

The upper sandy silt soils have low potential expansion with a measured Expansion Index of 22.

3.5 SOIL CORROSIVITY

Three samples of the upper soils were tested for pH, minimum resistivity, soluble chloride content and soluble sulfate content. The pH value ranged from 7.2 to 7.38. The minimum resistivity was between 800 and 1,250 ohm-cm. The soluble chloride content varied from 90 to 240 parts per million and the highest soluble sulfate content was 0.024% by weight.

3.6 SUBGRADE PERMEABILITY

The permeability test performed on the sample obtained from Boring A-13 resulted a low hydraulic conductivity (2.4×10^{-7} cm/sec). It indicates the on-site soils may not be adequate for dispersion of surface water. An additional permeability test is being performed on a sample obtained from Boring A-14 to confirm this conclusion.

3.7 SEISMIC HAZARDS

3.6.1 Faulting and Seismicity

The project site is in the seismically active Southern California region. Known regional active faults within 40 kilometers of the site that could generate significant ground shaking at the site include M. Ridge-Arroyo Parida-Santa Ana fault, Santa Ynez (West) fault, Red Mountain fault, Santa Ynez (East) fault, Ventura-Pitas Point fault, Los Alamos-W. Baseline fault, and Big Pine fault, among others. The closest of these is the M. Ridge-Arroyo Parida-Santa Ana fault located approximately 0.2 kilometer from the site.

The site is not located in a State of California Alquist-Priolo Earthquake Fault Zone. No known faults project into or cross the site.

3.6.2 Ground Motion

The site is likely to be subjected to strong ground shaking during the life of the proposed structures. To evaluate the ground motion and determine a peak level of ground acceleration that the site is likely to experience, a probabilistic seismic hazard analysis (PSHA) was performed using the computer program FRISKSP (Blake, 2000).

There are numerous attenuation relationships available for use in a PSHA. We used a combination of the Boore, Joyner & Fumal (1997), Bozorgnia, Campbell & Niazi (1999), Sadigh et al. (1997), and Idriss (1994) attenuation relationships included in FRISKSP for the probabilistic analyses.

The peak ground acceleration at the site for the Lower Level Earthquake (LLE), 50% probability of exceedance in 50 years, is 0.24g. For the Upper Level Earthquake (ULE) (i.e., Design Base Earthquake, DBE), 10% probability of exceedance in 50 years, the peak ground acceleration is 0.52g.

3.6.2 Liquefaction Potential and Seismically-Induce Settlement

The term "liquefaction" describes a phenomenon in which a saturated cohesionless soil loses strength and acquires a degree of mobility as a result of strong ground shaking during an earthquake. Using the deep boring data at the proposed building areas, we performed analyses of liquefaction potential and seismically-induced settlement for the project site under the Design Base Earthquake (DBE) based on the following parameters:

- An earthquake moment magnitude of 6.7;
- A peak ground acceleration of 0.52g; and
- A design groundwater depth of 19 feet.

The liquefaction potential of the site was evaluated using the procedures outlined by Seed et al. (1983) and updated by the NCEER workshop (Martin and Lew et al., 1999; Youd and Idriss et al., 2001). The seismically-induced soil settlements were estimated using the procedures outlined by Tokimatsu and Seed (1987). Results of the evaluations indicate the following conclusions:

(1) For the areas of Buildings "A" and "E", the liquefaction potential is low and the seismically-induced settlement is negligible.

(2) For the area of Building "B", the subsurface soils within the layer of silty sand from depths of approximately 25 to 30 feet are liquefiable when saturated. The seismically-induced soil settlement was estimated to be approximately 1.5 inches.

(3) For the areas of Buildings "C" and "D", the subsurface soils within the layer of silty sand from depths of approximately 19 to 20 feet are liquefiable when saturated. The seismically-induced soil settlement was estimated to be approximately 0.5 inches.

Based on the subsurface soil conditions, continuous liquefied layers are not anticipated to exist at the project site and seismically-induced soil settlement will only occur within localized zones.

3.6.3 Other Secondary Effects of Seismic Activity

The possible other secondary effects of seismic activity include tsunamis, flooding or seiches, landslides, and ground rupture. The potential threats from these secondary effects are discussed below.

Tsunamis: Tsunamis are tidal waves generated by fault displacement or major ground movement. The site is about three miles from the Pacific Ocean. Therefore, the possibility of damage from tsunamis is low.

Flooding or Seiches: Flooding may be caused by failure of dams or other water retaining structures due to earthquakes. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. There are no dams or other water retaining structures nearby this site. The potential for damage from seismically-induced flooding or seiches is low.

Landslides: The site has only minor relief. The probability of damage to the proposed construction as a result of seismically induced landslides is considered low.

Surface Fault Rupture: The site is not located in an Alquist-Priolo Special Studies Zone and no known active surface faults approach within 0.2 kilometer of the site. No ground rupture is expected.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL CONCLUSIONS

Based on our geotechnical investigation and seismic hazard evaluation study, we conclude that the proposed project is feasible from a geotechnical viewpoint, provided the recommendations contained in this report are implemented in the design and construction of the project.

4.2 EARTHWORK

Earthwork should be performed in accordance with the City of Santa Barbara Grading Ordinance, the City of Santa Barbara Municipal Code Section 28.87.250 titled "Development along Creeks", and the latest edition of the Standard Specifications for Public Works Construction (Greenbook, 2003). Excavations and cuts should be inspected during grading.

4.2.1 Site Preparation

Prior to construction, the site should be cleared of all vegetation, debris, loose soils, old foundations, and any other deleterious material.

4.2.2 Overexcavation and Recompaction

The entire footprint of every structure except Building "A" and to a distance of 5 feet beyond the structure should be overexcavated to a minimum depth of 2 feet below the existing grade or the finish subgrade elevation, whichever is lower, and replaced with engineered fill. For Building "A", heavy loads are anticipated. To reduce settlement, overexcavation for the footprint of Building "A" to a distance of 5 feet beyond the structure is recommended to be extended to a depth of 2 feet below the bottom of the foundation or a depth of 3 feet below existing grade, whichever is lower, and replaced with engineered fill.

The exposed bottom of the excavation must be inspected by the geotechnical consultant's representative, prior to placement of engineered fill, to ensure that competent bottoms have been exposed and that no additional overexcavation is necessary. Prior to placing engineered fill, the exposed bottom of overexcavation should be scarified to a minimum depth of 8 inches, moisture conditioned as necessary to achieve moisture content of 2 percent above optimum, and compacted in place to at least 90 percent relative compaction. Fill should be placed in loose lifts of 8 inches or less. Voids or holes resulting from the removal of trees and other structures should be overexcavated to a depth exposing firm and competent soil.

Parking pavement areas and concrete flatwork (such as slab-on-grade, sidewalks, hardscape, curbs, and gutters) should be underlain by a minimum of one foot of engineered fill or scarifying the exposed grade to a depth of 8 inches and in place processing, compacted to at least 90 percent relative compaction.

The on-site soils may be reused as compacted fill provided they are free of organics, deleterious materials, debris and particles over four inches in largest dimension.

4.2.3 Compaction Criteria

Unless stated otherwise, all fill should be compacted to a minimum of 90 percent relative compaction of the maximum density as determined by the ASTM D1557-91 test procedure. Compaction should be verified by observation, probing, and testing by a geotechnical consultant.

4.2.4 Import Materials

In general, import soils should not contain organic material, rocks greater than 4 inches in greatest dimension, debris and other deleterious materials.

Any import soils should be granular and non-expansive with an Expansion Index less than 30. All import soils, if used, must be tested and approved by the geotechnical consultant. Ideally, import soils should be tested and approved prior to delivery to the project site.

4.2.5 Temporary Excavations

Temporary excavations must be properly sloped or shored. If applicable, lateral loads due to surcharges from vehicle traffic or adjacent structures should be added in the shoring design. Excavated soil should not be stockpiled adjacent to the excavation.

Based on the earth materials encountered in our borings, excavation of 5 feet or less in depth may be performed with vertical sidewalls. Deeper excavation up to a depth of 15 feet can be accomplished in accordance with the OSHA requirements for Type C soils. Temporary cantilever shoring should be designed to resist a lateral earth pressure equivalent to a fluid density of 42 pound per cubic foot for level ground and exposed heights no greater than 15 feet.

The contractor is responsible for worker safety in the field during construction. The contractor shall conform to all applicable occupational safety and health standards, rules, regulations, and orders established by the State of California. In addition, other State, County, or Municipal regulations may supersede the recommendations presented in this section.

4.3 SLOPE STABILITY

Based on the configurations of the slopes along the western and southern boundaries next to the natural creeks, we performed engineering analyses to assess slope stability under both static and pseudo-static conditions. The analyses were performed for three typical cross sections (A-A', B-B', and C-C') as shown in Figure 2 using the computer program GSTABL7 with STEDwin (Gregory, 2003). The analyses were performed for a water level corresponding to a 100-year water surface elevation and also for a rapid drawdown condition.

The graphical outputs showing the input parameters and the resulting safety factors are included in Appendix D.

Under the static condition, the factor of safety for global stability is greater than 1.5. Under the pseudo-static condition with a seismic coefficient of 0.15, the factor of safety is greater than 1.1. Therefore, the slopes satisfy the minimum requirements for global slope stability and the global instability is not expected.

To reduce the likelihood of surface erosion for the slopes, it is recommended that diversion of water from the slope face be implemented.

4.4 SEISMIC DESIGN PARAMETERS

The project site is not in a currently designated Alquist-Priolo Earthquake Fault Zone as defined by the State of California. However, strong ground shaking due to seismic activity can be expected at this site in the future. Based on the California Building Code (CBC, 2001), the site is assigned to Seismic Zone 4, soil profile S_E. The nearest seismic source type A fault is the San Andreas fault located about 64 kilometers from the site. The nearest type B fault is the M. Ridge-Arroyo Parida-Santa Ana fault which is approximately 0.2 kilometer away from the site and has a magnitude of 6.7.

In accordance with the 2001 CBC, the proposed structures can be designed using the seismic design parameters listed in Table 2. A design response spectrum can be developed using the seismic design parameters and Figure 16-3 of the 2001 CBC.

TABLE 2. CBC SEISMIC PARAMETERS

Seismic Zone Factor Z	0.4
Soil Profile Type	S_E
Seismic Source Type	B
Near-Source Factors	$N_a = 1.3$, $N_v = 1.6$
Seismic Coefficients	$C_a = 0.47$, $C_v = 1.54$
Control Periods	$T_s = 1.313$, $T_o = 0.263$

4.5 FOUNDATIONS

4.5.1 Foundation Type

Conventional spread or continuous footings may be used to support the proposed structures. Precautions should be taken to prevent the entrance of surface water to the soils supporting the footings.

4.5.2 Foundation Design

For square footings, they should have a minimum embedment of 2 feet below surrounding lowest finished grade and a minimum width of 2 feet. The square footing with the recommended minimum sizes may be designed for a net allowable vertical bearing pressure of 3,000 psf for dead-plus-live loads. The allowable bearing pressure of footing may be increased by 100 psf for each additional foot of foundation width or by 700 psf for each additional foot of foundation depth of embedment, up to a maximum allowable bearing pressure of 5,000 psf.

For continuous footings, they should have a minimum embedment of 2 foot below surrounding lowest finished grade and a minimum width of 1.5 feet. The continuous footing with the recommended minimum sizes may be designed for a net allowable vertical bearing pressure of 2,500 psf for dead-plus-live loads. The allowable bearing pressure of footing may be increased by 100 psf for each additional foot of foundation width or by 700 psf for each additional foot of foundation depth of embedment, up to a maximum allowable bearing pressure of 3,500 psf.

The above bearing pressures may be increased 33% when considering temporary forces such as seismic or wind.

4.5.3 Settlement

The maximum settlement for a footing supporting a 160-kip column load at Building "A" is about 1 inch with a maximum differential settlement of about 0.5 inch. The maximum settlement for a footing supporting a 90-kip column load at the other buildings is about 0.6 inch with a maximum differential settlement of about 0.3 inch.

The maximum settlement of a continuous footing is about 0.5 inches with a maximum differential settlement of about ¼ inch.

4.5.4 Lateral Resistance and Earth Pressures

For design, resistance to lateral loads may be assumed to be provided by friction acting on the base of the footings and floor slab, and by passive earth pressure on the sides of the foundations. A coefficient of friction of 0.4 may be assumed with the dead load forces. An allowable passive earth pressure of 250 psf per foot of depth up to a maximum of 3,000 psf may be used for the sides of footings poured against undisturbed native soils or properly compacted fill. The value of the allowable passive earth pressure includes a factor of safety of 1.5. The allowable passive pressure may be increased 33% when considering temporary forces such as seismic or wind loadings.

Static active earth pressure for retained native soils (sandy silt) should be 42 psf per foot of depth. Where non-expansive soils are used as backfill the active earth pressure may be reduced to 35 pcf. At-rest earth pressures should be 62 pcf and 53 pcf for native soils and imported backfill, respectively.

4.6 SLABS-ON-GRADE

The floor slabs need to support a slab loading of 200 psf. Slabs-on-grade should be underlain by a 6-inch thick layer of free-draining granular materials that should be moisture conditioned to 2 percent above optimum moisture content and compacted to 90 percent of the maximum dry density as determined by the ASTM D1557-00 test method.

The slabs may be designed using a subgrade modulus of 50 pounds per cubic inch. A minimum of 6-by-6-inch No. 10 wire mesh or equivalent reinforcement should be used in slabs-on-grade. The subgrade should be maintained in a moist condition until the floor slab is poured.

If a moisture sensitive floor covering such as vinyl tile is used, slabs should be underlain by a 6-mil-thick polyethylene plastic vapor barrier. If the barrier is used, it should be covered with 2 inches of sand to prevent punctures and to aid in concrete curing. Joints should be lapped at least 6 inches and taped.

4.7 SURFACE DRAINAGE

Inadequate control of run-off water and/or heavy irrigation after development of the site may lead to adverse water conditions. Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture related problems and differential movements from soil heave/settlement.

Surface drainage should be carefully taken into consideration during grading, landscaping and building construction. Positive surface drainage should be provided to direct surface water away from structures and toward the street or suitable drainage devices. Ponding of water should not be allowed. Paved areas should be provided with adequate drainage devices, gradients and curbs to reduce run-off flowing from paved areas onto adjacent unpaved areas.

4.8 PAVEMENTS

We have designed pavement sections for a Traffic Index (TI) of 5.2 for light duty traffic and 6.3 for heavy-duty truck traffic. As directed by the designer, the loading dock area on the south side of the project site were designed for a TI of 8.

Both asphaltic concrete and Portland cement concrete were considered. The design depends upon vehicular traffic, strength of the pavement materials and the subgrade soils.

The near surface soil has a measured R-value of 54. An R-value of 30 was used in our design following the design procedure of Caltrans (1995).

Flexible pavements consisting of asphalt concrete (AC) over Class 2 Aggregate Base (AB) (i.e. composite sections) or full-depth asphalt concrete (AC) over the native soils are recommended. Table 3 presents the recommended flexible structural sections.

TABLE 3. RECOMMENDED FLEXIBLE PAVEMENT STRUCTURAL SECTIONS

Traffic Index	Flexible Pavement Section Thickness (inches)	
	Composite	Full-Depth
5.2	3-inch AC/6-inch AB	6-inch AC
6.3	4-inch AC/7-inch AB	7-inch AC
8.0	5-inch AC/11-inch AB	10-inch AC
Notes: AC = Asphalt Concrete; AB = Aggregate Base (Class 2)		

Because the subgrade soil has relatively high R-value, rigid pavements consisting of Portland Cement Concrete Pavement (PCCP) over AB can be used as an alternative to the flexible pavements. Table 4 presents the recommended rigid structural sections.

TABLE 4. RECOMMENDED RIGID PAVEMENT STRUCTURAL SECTIONS

Traffic Index	Rigid Pavement Section Thickness (inches)	
	PCCP	AB
5.2 and 6.3	6	4
8.0	6	6
Notes: PCCP = Portland Cement Concrete Pavement; AB = Aggregate Base (Class 2)		

According to the designer, permeable pavers over AB were also considered. Our recommended thickness of AB is listed in Table 5.

TABLE 5. RECOMMENDED AB THICKNESS FOR PAVERS

Traffic Index	AB Thickness (inches)
5.2	12
6.3	14
8.0	18
Note: AB = Aggregate Base (Class 2)	

All pavement construction should be performed in accordance with the Standard Specification for Public Works Construction (Greenbook, 2003). Field observation and periodic testing, as needed during placement of base course material, should be undertaken to confirm that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soils should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction. Aggregate base should be placed in thin lifts, moisture-conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

4.9 CEMENT TYPE AND CORROSION MEASURES

Based on the measured soluble sulfate content (see Appendix C) and Table 19-A-4 of CBC (2001), sulfate resistant cement is not required for concrete in contact with on-site soils. Type I or Type II Portland cement is recommended. However, the chloride content (90 to 240 ppm) is relatively high and resistivity (800 to 1,250 ohm-cms) is relatively low; thus the on-site soils are corrosive to buried ferrous metals. Corrosion mitigation measures, such as the following, are recommended:

- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

4.10 UTILITY TRENCH BEDDING AND BACKFILL

Bedding can be defined as the material supporting, surrounding and extending 1 foot above the top of a pipe. Bedding must be sand, gravel, crushed aggregate or free-draining granular material having a sand equivalent (SE) of at least 30. The onsite soil is not suitable for use as bedding. Soil used for bedding must be approved by the geotechnical consultant prior to importation to the site.

Bedding must be placed on a firm and unyielding subgrade so the pipe is supported for the full length of the barrel. The trench bottom must be inspected prior to placement of bedding material to insure that a firm and unyielding subgrade is exposed. If the subgrade is loose or unstable, the unsuitable subgrade soil must be overexcavated and replaced with compacted bedding material. Bedding must be placed uniformly on each side of the pipe and compacted to at least 90 percent relative compaction in accordance with ASTM D1557. Bedding placement must conform to the Greenbook.

4.11 REVIEW OF CONSTRUCTION PLANS

Recommendations contained in this report are based on preliminary plans. The geotechnical consultant should review the final construction plans and specifications in order to confirm that the general intent of the recommendations contained in this report have been implemented into the final construction documents. Recommendations contained in this report may require modification or additional recommendations may be necessary based on the final design.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

It is recommended that all grading, excavation, and installation of foundations be performed under the inspection and testing of the geotechnical consultant during the following stages of construction:

- Grading operations, including overexcavations and placement of compacted fill;
- Removal of existing pavement structural sections, curb and gutter;
- Preparation of pavement subgrade;
- Placement of aggregate base;
- Footing excavations;
- Excavations and backfilling for utility trenches; and
- When any unusual subsurface conditions are encountered.

5.0 CLOSURE

This report is intended for the use of Regency Centers and its consultants for the design of the proposed shopping center located at 3759-3763 State Street in the City of Santa Barbara, California.

The findings and recommendations contained in this report are based on the results of the field investigation, laboratory tests, and engineering analyses, combined with an extrapolation of subsurface soil or rock conditions between and beyond the boring locations.

Services performed by Arroyo Geotechnical have been conducted in accordance with generally accepted professional geotechnical engineering principles and practices at this time. No other representation, express or implied, and no warranty or guarantee is included or intended.

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